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PRELIMINARY GEOMECHANICAL ASSESSMENT OF THE MONTAUBAN MINE

M.C. Bétournay

Rock Mechanics Laboratory

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by
M.C. Bétournay *

ABSTRACT

At the request of the Montauban mine, Quebec, a field visit by the CRM and CANMET was performed to provide a preliminary assessment of the planned mining strategy. The plan is to use open stoping, 45 m wide stopes with 6 m pillars oriented on dip. There will be a series of thirteen such stopes along the strike of the ore zone. No backfill is intended.

The mining will be affected by three structural features: subvertical joints of varying extent and concentration, well formed but irregular schistosity and shear zones. Application of the NGI rock mass classification indicates that the south sector is of poor quality, the north sector of fair quality. The mine's support measures under these conditions are adequate, but when the full spans are opened, longer, tensioned bolts would be required to prevent serious hangingwall degradation. It is estimated that the mine workings will be experiencing higher stress concentrations if open stoping is used, due to the large spans, narrow pillars and consecutive openings. Orienting the pillar support along the strike will provide less stress concentration on the openings; to maintain the same extraction ratio, the integrity of the openings will depend on a single pillar.

Being at an early stage the mine should embark on a information gathering campaign: coring, structural geology, strength and deformation properties and hangingwall monitoring. This would give the operators a greater appreciation for local conditions. It would also supply data for numerical modelling, which, in the case of the Montauban mine, is highly recommended to provide pertinent stability assessment under the planned strategy and existing structural setting.

*Physical Scientist, Mining Research Laboratories, CANMET, Energy, Mines and Resources Canada.

keywords

stability assessment, open stoping, rock mass quality, support requirements, elastic analysis, pillar strength, geological structure, stope span, numerical modelling, multiple anchor extensometer monitoring.

ÉVALUATION PRIMAIRE GÉOMECHANIQUE DE LA MINE MONTAUBAN

par
M.C. Bétournay *

RÉSUMÉ

Suite à la demande de la mine Montauban, Québec, une visite de site par le CRM et le CANMET a été faite pour fournir une évaluation préliminaire de la stratégie minière envisagée. La stratégie est d'utilisée une méthode de minage genre chambre ouverte, avec des étendues de 45 m séparées par des piliers de 6 m orientés sur le pendage. Il y aura une série de 13 chambres placées sur l'azimuth du gisement. Aucun remblai n'est planifié.

Le minage sera affecté par trois structure géologiques: des diaclases subverticales de grandeurs et concentrations diverses, une schistosité bien formée mais irrégulière, et des zones de cisaillements. L'application de la méthode NGI de classification des massifs indique que le secteur sud est de pauvre qualité, le secteur nord de qualité passable. Sous les conditions minières actuelles, les méthodes de soutènements utilisées par la mine sont adéquates, mais lorsque les chambres seront ouvertes à leur pleines grandeurs, des boulons plus longs, tensionnés, seront requis pour éviter une dégradation sérieuse de l'éponte supérieure. Il est estimé que des ouvertures non-remblayées auront plus de concentrations de contraintes, à cause de leur grandeurs élevées, grosseurs restreintes de piliers, et le nombre d'ouvertures consécutives. L'orientation des piliers parallèle à l'azimuth du gisement réduira les concentrations de contraintes sur les ouvertures; pour maintenir le même taux d'extraction, la stabilité de ces ouvertures ne dépendra que d'un pilier.

La mine, étant toujours à un stage primaire, les dirigeants devraient adopter une campagne de ceuilletes de données: carottage, géologie structurale, propriétés de déformation et de résistance, et suivi de l'éponte supérieure. Ceci produirait une plus grande appréciation de la part opérateurs pour leurs conditions locales. Ces informations pourraient également alimenter la modélisation numérique qui, dans le cas de la mine Montauban, est fortement recommandée pour fournir une évaluation de stabilité basé sur la stratégie minière de la mine et la géologie structurale existante.

*Recherchiste en Sciences Physiques, Laboratoires de Recherches Minières, CANMET, Énergie, Mines et Ressources Canada.

mots clés

évaluation de stabilité, chantier ouvert, qualité du massif rocheux, exigences de soutènements, analyse basé sur l'élasticité, résistance des piliers, structures géologiques, étendue du chantier, modélisation numérique, suivie d'ouverture par extensomètre à ancrage multiple.



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1. INTRODUCTION

In July of 1986 Mr. Frank Johnson, Manager of the Montauban mine, Quebec, contacted CANMET and requested a field visit. The visit was intended to provide a preliminary technical assessment of the operation's mining strategy, under current ground conditions. The mine called upon CANMET because of its expertise in a similar mine setting, the Elliot Lake uranium mines. Accompanying CANMET was the CRM (Centre de Recherches Minérales). The visit was carried out by Messrs. Robert and Bienvenu of the CRM and by Dr. Vongpaisal and the author, from CANMET.

2. MINING BACKGROUND

The property is located some 50 km south-west of Quebec City, on the fringe of the Canadian Shield, where the latter meets the St. Lawrence Lowlands sedimentary rock sequence (figure 1). There, Muscocho Exploration operates the Montauban mine which excavates ore grading 0.137oz gold and 1.833oz silver per ton. The mine has been in full operation since 1983 at a 400 tpd production rate. The activity is now in the property's second ore zone, the first having been mined out. This zone has a mine life of 4.5 years.

The ore zone is situated in metamorphic rock, on the limb of a syncline oriented NW-SE. It is defined by one tabular, gently dipping horizon. Its thickness is variable, 1.8-2.5 m, within which gold is finely disseminated. The footwall and hangingwall consist of a well banded quartz-mica schist, figure 2. The ore zone becomes increasingly laminated towards the southern portion of the mine, figure 3.

The basic mining strategy is to create a room and pillar system, figure 4. Because fill would prove uneconomical, the pillars and roof bolting are expected to provide the necessary, permanent support. The planned pillar dimensions are 6m wide by 45m long. The stope dimensions planned are 45m in width and in length. With the pillars and the excavated stopes paralleling the dip, the setting would be similar to mining at Elliot Lake.

The mine will operate on two levels. The main haulageway, with loading points for trucks, is located some 60 to 120 m below surface depending on surface topography and haulageway grade. Located some 15m above is the sublevel, accessible from the haulageway by several raises. The stopes are planned to open up dip from the sublevel. Once blasted, the ore is scraped back to the stope's mill hole (where it will be loaded on the trucks). The ore is trucked to surface by the ramp, figure 5; there is no shaft. The mine has completed haulageway and sublevel development and started excavating some stopes.

3. UNDERGROUND VISIT

The underground tour included inspection of the ramp, part of the haulageway and most of the sublevel (120S to 495S).

The rock mass at the mine contains three structural features:

- 1) Localized joints
- 2) Schistosity
- 3) Shear zones

Two families of joints were identified. It must be underlined here that each set occurs in a localized fashion rather than evenly distributed. The remainder of the rock mass is intact.

A minor family trending north and dipping 45° west is subparallel to the ramp. A subvertical set (figure 6), trending east-west and dipping south, is seen in several areas of the mine. However, the distribution and extent of the joints are uneven. In the sublevel, the general trend is for the joints to shorten and diminish in density from the south to the north. From 495S to 360S, the joint length is extreme, >5m, and the joints appear regularly spaced, 2 to 3m intervals. In the mid area, 360S to 260S, the joints are not as extensive, 2 to 5m, and tend to group locally. The northernmost area seen, 260S to 100S, contain few subvertical joints, usually <2m long.

The joints are smooth and planar. Most are closed but it is apparent that some sub-verticals have been opened prior to excavation, at least long enough to have circulating groundwater deposit silt and sand, figure 7. These infillings no doubt originate from surface overburden.

The second major structure present is the schistosity, occurring in the hangingwall and footwall. When banding is present in the ore zone it parallels the schistosity. Ore zone banding is prevalent in the southern and mid region and diminishes in importance moving to the north to the extent of disappearing.

The schist is not regular in the thickness of its component bands, nor is it in the concentration of mica. The greater concentration of mica causes the rock to detach in slabs, 20 to 60 cm thick. This structural weakness was responsible in July of 1986 for the fall of a large block from the roof, causing one fatality.

The third structural element are shear zones. Two were seen, one crossing the ramp

at the powder magazine, the other on the haulage level at 120S. Both were extensive, $> 5m$, vertical and parallel to the direction of subvertical jointing. Each zone consists of a central, closed continuous slip plane with lateral discontinuous fissures on either side of the slip, figure 8.

The present method of support in the sublevels is 1.5m resin grouted mechanical bolts in a $1.2m \times 1.2m$ or $1.5m \times 1.5m$ pattern. Those applied immediately after an opening is created are holding the back very well. Where there has been a delay in installation, detachments have occurred, figure 2. Posts have been used locally with success to remedy this situation.

4. ANALYSIS

This report is based on a short underground visit. Because the structural elements are so well defined and few, a preliminary geomechanical assessment is relevant. However, more intensive field and lab work are necessary to produce a more accurate assessment that the mine could use in establishing a long term mining strategy. The last chapter will outline some of the future work recommended.

4.1 Mining Strategy

Room and pillar is the mining planned mining method. It is the most economical. Variations are acceptable to the mine, as long as stability and recovery are not affected. Possible variations are:

- pillar orientation; on strike or as now envisaged, down dip.
- opening size, pillar size; ideally, the mine wishes to extract as much ore as possible, the reason for the 45m opening span and the 6m pillar width.
- pillar location to accomodate structure

4.2 Rock Mass Quality and Support Requirements

The important structural elements to include in the general strategy are the subvertical joints and the schistosity. Shear zones are localized phenomena.

The schistosity is an element that will require constant support and attention. In narrow orebodies with planes of weaknesses, stress concentrations and /or large spans can facilitate roof tensile cracking, roof parting or floor heave (1). Because the presence of

parting planes in a roof cannot always be predicted, post-blast scaling and roof bolting should be done before production drilling is resumed.

The subvertical joints will affect the openings in two ways. If included in down-dip pillars they will cause pillars to split and weaken. Located in large roof spans, they will facilitate roof tensile cracking and, when they are numerous and well grouped, can combine with the schist to facilitate block formation. The worst mining conditions are thus expected in the south portion.

The shear zones will cause problems irrespective of location: pillar splitting, roof degradation and vertical offsetting.

A bolting system which includes sufficient pre-tensioning and anchoring depth, will keep roof laminations tied together. The resulting laminated beam will have greater strength than individual parting layers. In the case of potential block formation, the tensioned bolts will maintain a tight interlocked mass. However, while in most average spans such interlocking blocks can form efficient arching, it is unlikely that efficient arching can be obtained in stopes with large spans.

Ground control can also be helped by using careful blasting. Firstly, the back will be most stable when its outline is parallel to the schistosity. Secondly, to prevent roof overbreak and strata loosening as well as preventing pillar damage, post split blasting techniques can be employed.

The NGI rock mass classification system (2) is applied to estimate empirical support requirements and provide a qualitative description of the rock mass quality. The two extremes in ground quality (south and north portion) are considered based on the field visit. The formula to obtain the quality factor "Q" is:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where

RQD is the rock quality designation

J_n is the number of joint sets

J_r is the roughness of the joints

J_a is joint alteration

J_w is the water condition on the joint

SRF is the stress reduction factor

An estimate of the RQD is made using the field formula:

$$RQD = 115 - 3.3J_v$$

J_v is the number of joints per meter cube. Respectively, RQD in the south and north sectors is 95 and 100 (J_v of 7 and 5) respectively. As well, the following parameters are used:

south sector

$J_n = 4$ (this if for two joint sets, subvertical & schistosity)

$J_r = 1.0$ (for smooth, planar joints)

$J_a = 4.0$ (given to represent the soft micaceous coating on the schistosity)

$J_w = 0.66$ (given to represent occasional inflow of water, as described by the mine)

$SRF = 1.0$ (medium stress)

$$Q = 3.8$$

north sector

All variables are the same as the south sector except for J_n , here it is 3 (for one joint set, schistosity, and one random, the subvertical family)

$$Q = 5.4$$

Figure 9 shows that qualitatively, the south sector falls in the poor NGI category, the north sector in the fair NGI category.

To relate the quality factor to support requirements, D_e , the equivalent dimension factor is used.

$$D_e = \frac{\text{span}}{\text{support ratio (ESR)}}$$

For a permanent opening, $ESR = 1.6$; with a span of 45 m

$$D_e = 28.75$$

Again referring to figure 9 and table 1, the NGI support category for the south portion is a bolting pattern of 1 to 1.5 m and secondary reinforcement such as mesh reinforced shotcrete. The bolts have to be tensioned to a suggested pressure of 1.5 kg/cm^2 .

The north sector support category is a bolting pattern 1 to 2 m and secondary reinforcement such as mesh reinforced shotcrete. The bolts have to be tensioned to a suggested pressure of 1 kg/cm^2 . The roof bolt length is calculated by:

$$\text{length} = 2 + 0.15 \frac{\text{span}}{\text{ESR}}$$

For the south and north sector

$$\text{length} = 6.2 \text{ m}$$

Using as an example the widest span, 20 m, presently occurring at the mine, this system would recommend bolt lengths of 3.9 m. It is apparent that this estimation errs on the conservative side since the 1.6 m bolts installed there by the mine are sufficient, figure 10. The 6.2 m value must then also be regarded as an overestimation. However, it is expected that under 45 m spans a 1.6 m bolt length will not be adequate. The mesh reinforced shotcrete would also appear to be an overestimation; except for blocky ground conditions where screening is expected to be a necessary adjunct to bolting. Bolt lengths will have to be long enough to remain anchored and with enough carrying capacity to support the weight of a loose block. There should be one bolt to support each large block. A future study using numerical modelling will be able to provide an evaluation of which support method to use, for both mass qualities.

If a bolt length is required to be longer than the footwall to hangingwall opening (1.8-2.5 m), only the flexible cable bolt can be installed because of space limitation.

4.3 Stress Effects on Pillars and Openings

The main features to consider in analysing stress effects are: direction and magnitude of in-situ principal stresses, orientation of the opening within the stress field and opening geometry.

The proposed stope and pillar layout at the Montauban mine is simple. Thirteen consecutive 45 m stopes will each be separated by 6 m pillars. Only a central deviation prevents this arrangement from being linear, figure 2. Another alternative, while still maintaining the same extraction ratio, is to orient a 6 m pillar along strike, creating two long stopes each 22.5 m wide, measured down dip. In either case, stress concentration and deformation in this multiple opening setting would be accurately modelled by computer methods, discussed in chapter 5.

Being near surface it is possible that gravity loading predominates. In this case, the tributary area theory (3) identifies the average pillar stress as

$$\sigma_p = \frac{S_o A_t}{A_p}$$

where

S_o =uniaxial field stress

A_p =cross sectional area of the pillar

A_o =cross section tributary extraction area

$$A_t = A_p + A_o$$

In the case of inclined pillars, S_o must be converted to reflect horizontal and vertical in situ stress components.

The Canadian Shield possesses ground stresses larger horizontally than vertically. Herget (4) has established that the principal stresses are oriented about parallel and at right angles to the earth's surface. The following relationships were calculated:

$$\sigma_{vertical} = 0.0260 \text{ to } 0.0324 \text{ MPa per metre depth}$$

$$k_{max} = \frac{\sigma_{horizontal \max}}{\sigma_{vertical}} = \frac{253.87}{\text{depth (m)}} + 1.45$$

$$k_{min} = \frac{\sigma_{horizontal \min}}{\sigma_{vertical}} = \frac{279.72}{\text{depth (m)}} + 0.88$$

The major principal stress is reported to be oriented NE-SW to E-W. Thus the major principal stress (horizontal) is oriented roughly parallel to the down-dip pillars, case 1. In case 2, an on-strike, 6m pillar is oriented orthogonally to the major principal stress.

For case 1, the effects of the vertical and horizontal stresses (table 2) are

$$S_o = \sigma_v \cos^2 \alpha + \sigma_h \sin^2 \alpha$$

where

α =orebody dip, 20° at Montauban

Based on Herget's values, $S_o=4.0$ MPa. The pillar load according to the tributary area theory in case 1 is thus 45 MPa; in case 2 where the pillar is along strike (not inclined), the pillar load is 35 MPa.

The highest pillar loads would be anticipated if the superposition effects of stopes on narrow pillars and the effects of wide spans were taken into consideration. A simple analysis to evaluate the stress concentrations existing in this situation is provided by two-dimensional elastic analysis. Here the approach by Geldart and Udd (5) will be used in identifying the effects on a single opening:

The tangential stress imposed on the opening surface is given by:

$$\frac{\sigma_{\eta}}{S_y} = \frac{2v(1+k) + (1-k)(1-v^2)\cos 2\beta + (1-k)(1+v)^2\cos 2(\beta-\eta)}{(1+v^2) + (1-v^2)\cos 2\eta}$$

where

σ_{η} = tangential stress on the ellipse surface

v = semi-major axis / semi-minor axis = axis ratio

$k = S_x/S_y$ = pre-mining horizontal / vertical stress = stress ratio

β = angle clockwise from semi-major axis to S_x

η = elliptical co-ordinate measurement clockwise from the semi-major axis

Stress concentration calculations, figures 11-12, table 2, are based on Herget's values. These are averages only. Local discrepencies could create higher or lower values. A depth of 120 m has been chosen, as worst possible case; the shallower openings will witness lower loading.

Under this analysis, the opening in case 1 will impose a very high stress concentration on the abutments, more than in case 2. The superposition of stresses from adjoining stopes, because of the narrow pillars, will magnify the load distribution calculated for a single opening. The thirteen consecutive openings in case 1 will also magnify the load distribution.

Thus the 45 to 117 MPa range seems likely as possible pillar loads for case 1, 35 to 75 MPa range for case 2; this is for a pillar at 120 m depth.

It is essential that these pillars be strong enough to fulfill the role expected of them. This could be permanent support or gradual pillar failure after mining has been finished.

Pillars fail in several fashions; if the pillars are massive enough, rather than by weakening along structure planes, high compressive stresses can lead to splitting, hourglassing or crushing (if the rock is of high strength). The high values of compressive stress possible in the pillar-hanging wall, -footwall corners may impose shearing movements along the schistosity planes. The mine, however, cannot afford pillar degradation during mine life (or none at all if it leads to surface subsidence where buildings can be damaged) under such large spans; it would lead to serious loss of support capacity. Case 2 has the drawback of having only one pillar for permanent support. Akin to this limitation, the mine cannot

afford to carry out actual pillar failure tests to determine pillar strength. Instead it has to depend on lab tests on various specimen sizes and back-analyse stable pillars to provide strength values. The serious drawback with this method is that lab samples provide a higher strength than actually encountered in the field.

This problem was addressed successfully by Hedley (6) who performed lab tests on specimens of various sizes and extrapolated the results to pillar size to obtain typical strength values for Elliot Lake mine pillars. He also combined this to back analysis of stable and unstable mine pillars. This information was then incorporated into a safety factor analysis between strength and induced stress, producing design curves for pillars. Such design could also be applied for the Montauban mine, once accurate in-situ stress determinations are made.

With elastic analysis, compressive strength values obtained for roof mid-span will ensure that tension cracks and parting of the schistosity (and subverticals in case 1) will not occur.

5. FUTURE STUDIES

The mine is still at an early production stage. It is thus important to obtain as much pertinent information as possible to examine a variety of mine layouts and mining sequences and arrive at a stable and economic mining strategy. Since the mine's extraction ratio is high, $\approx 90\%$, the loss of a pillar cannot be afforded.

The recommended components for such a study are:

1. Coring and structural data gathering
2. Strength determination and deformation
3. Numerical modelling
4. Site monitoring to be done in large openings during steps 1-3, initially for data input and subsequently for monitoring and model refinement.

Comprehensive core drilling and structural data gathering programs are needed to evaluate the local and general rock quality. This would identify support requirements, pillar orientation/location and provide the operating personnel with a "feel" for the rock mass in which they are operating.

The core will be used to test for rock strength and deformation characteristics required in evaluating pillar support capabilities and roof/floor behaviour. Pillars at shallow

depth can be effective means of support. The strength carried by pillars, when beyond supporting gravity load only, is difficult to assess. Just as difficult is the value to adopt for design of openings, from laboratory test on specimens such as rock core. There exists several empirical approaches to transpose lab results. A choice of which to adopt is easier when numerical modelling and further evaluation of structure and mass quality are done. As well, the structural record in the core will provide joint characteristics to complement structural geology field work and thus arrive at values to be used in the NGI classification. The NGI support recommendations do not seem apt to be used here.

To represent variations in mass properties, it is important that core be obtained from each different rock mass sector, and from the footwall, hangingwall and ore zone. If core was obtained from the hangingwall, a general impression of the surrounding schistosity will provide information to the numerical modeller in the form of general weakness trends and local considerations. Furthermore, tensile testing of specimens from the back (and the floor, if deemed necessary) will provide additional basic data input for modelling. Identification of support requirements is thus also facilitated.

Because the structural elements are so well defined, the exact location, attitude and extent of each joint of importance ($>0.5\text{m}$ in this case) and shear zone can be easily obtained. The location and trace should be plotted on plans and sections along with the orientation values. This will help the mine staff and modellers in deciding on pillar dimension and location.

In a future study of the Montauban mine, finite element modelling will be a very important element of design, in determining stress concentrations and the stability of openings in multiple opening settings. It can well represent the layered nature of the schistosity and address itself to the surrounding rock mass, not just the opening boundary. It is a flexible method in that various mine plans and geometries can be modelled and evaluated. By studying various models, the best mining sequence to avoid high stress concentrations and ground control problems can be established.

Based on modelling results, the mine can perform a cost analysis on the economics of varying pillar size *vs.* artificial support costs.

In situ stress measurements would reveal the orientation and magnitude of the major principal stresses. This would provide the precise values to use in modelling and other analyses. Though average values can be used, this could lead to over/underestimation of loads by assuming no anomalous stress conditions exist at the mine site.

Roof monitoring in such geological material is important. Roof stability is necessary

for two reasons: to prevent dilution by waste rock and to maintain the stope in operating condition. The relative displacements of selected points in a borehole oriented normal to the roof surface are required. Extensometers with multiple anchors would be adequate. The interpretation of data needs to consider the geological features seen in the core log. The instrument can be installed in the holes that provided the rock core. Extensometer location should be in the midspan area of the stope hanging wall. Floor to ceiling convergence measurements will not be adequate due to the activity of the ore scrapers.

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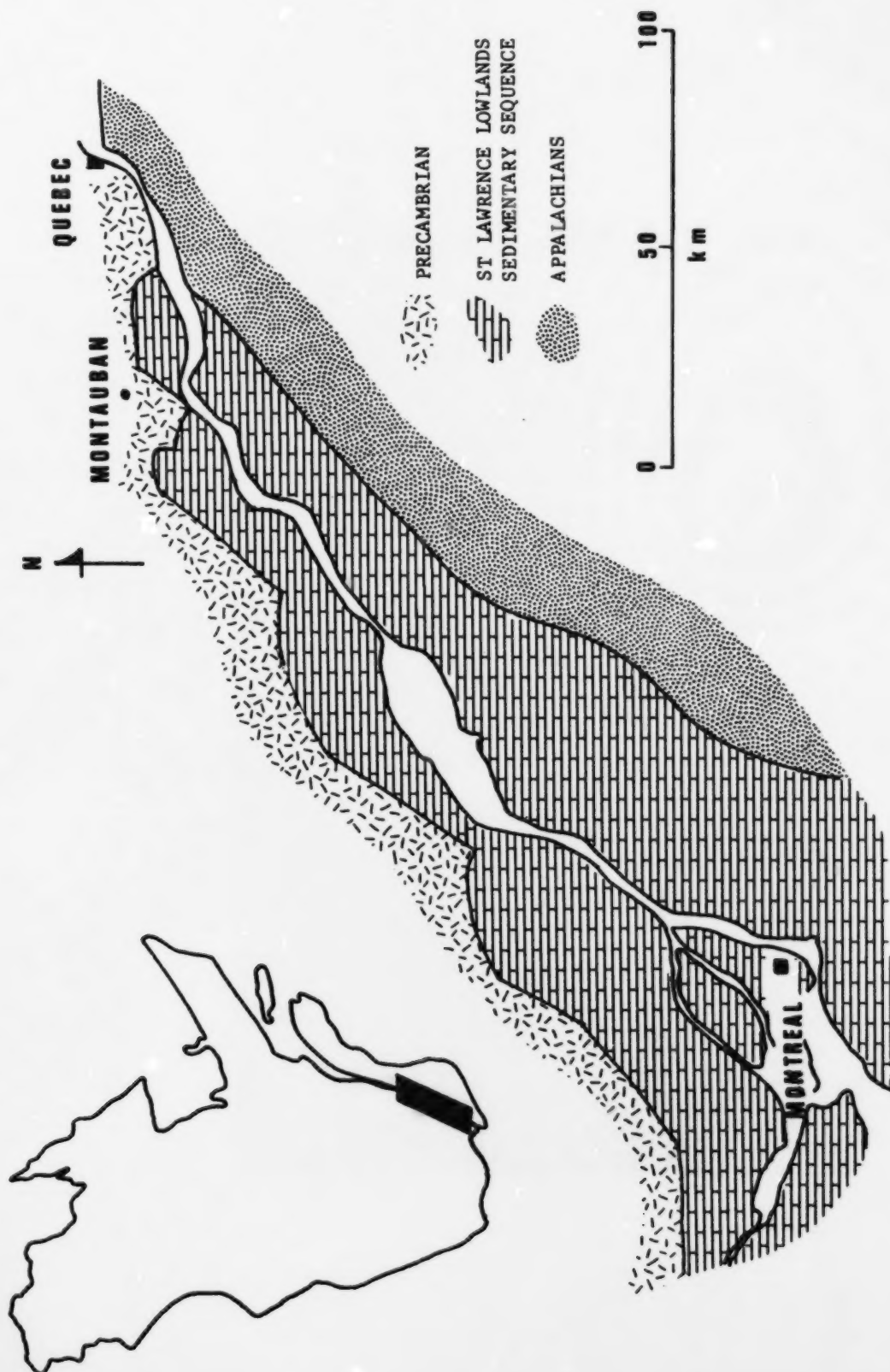


Fig. 1 - Montauban mine location.



Fig. 2 - Well developed schistosity in the hangingwall of the Montauban mine. Note the irregular banding thickness and parting of the roof due to delayed bolt installation. The post is in place to provide remedial action.



Fig. 3 - Well laminated ore zone, south sector;
the hangingwall schistosity is also apparent.

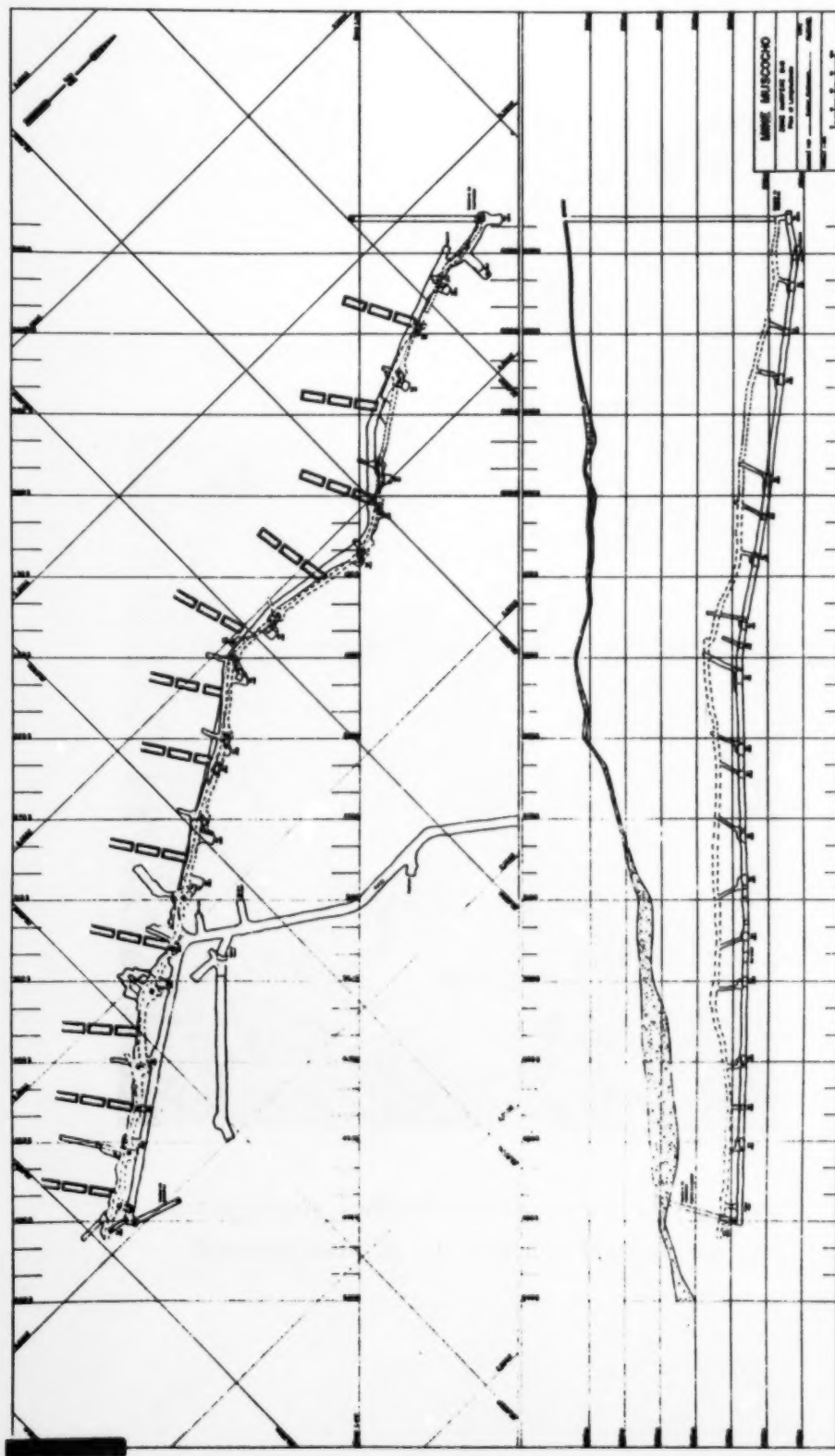


Fig. 4 - Montauban mine plan and section.



Fig. 5 - Montauban mine entrance.



Fig. 6 - Extensive, closed subvertical joints grouped next to the powder magazine.



Fig. 7 - Opened subvertical joint with silty-sand infilling.



Fig. 8 - Shear zone in the haulageway level at 120S. Looking upward.

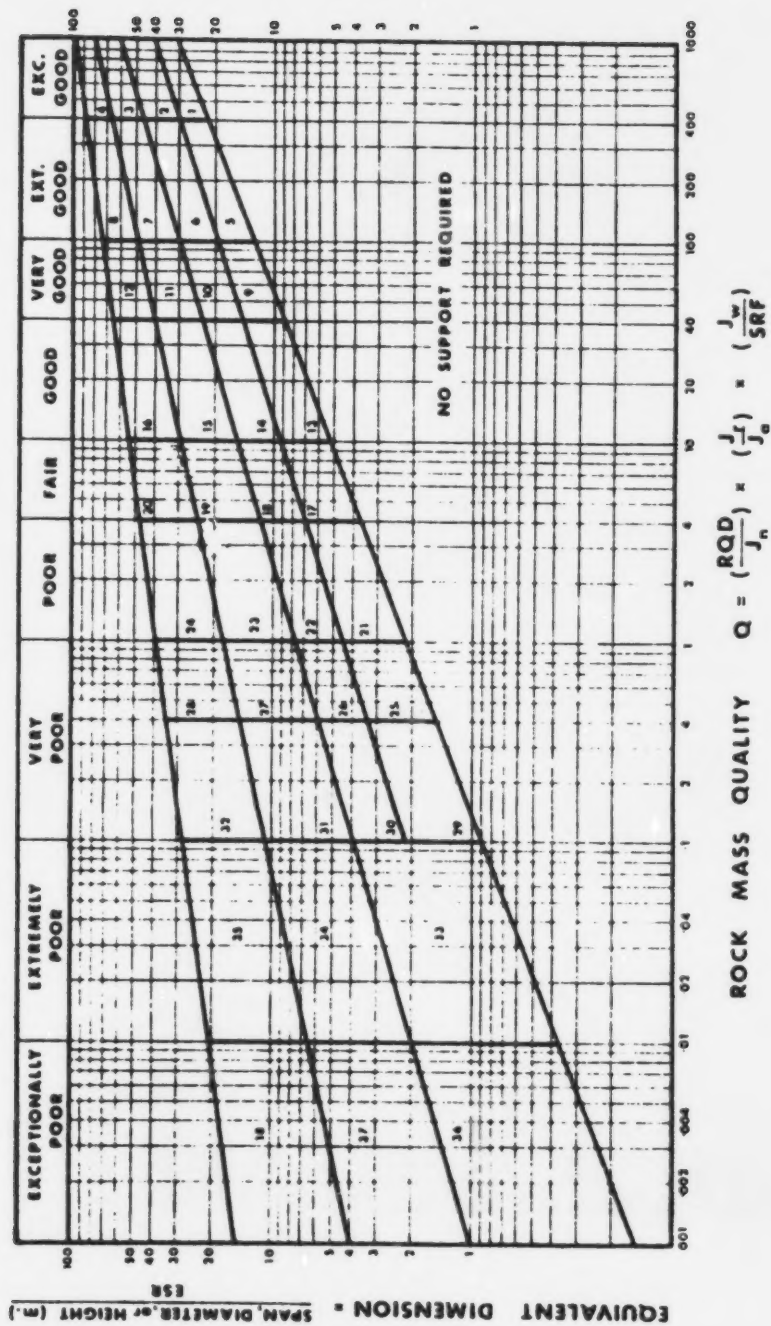


Fig. 9 - Tunnel support chart with 38 categories of support. From Barton et al. (1).



Fig. 10 - Stoping at the Montauban mine, the back is parallel to the schistosity. Looking updip. Here, there was no delay in bolting. The bolting pattern is roughly 1.2mx1.2m. Note the sub-vertical joints.

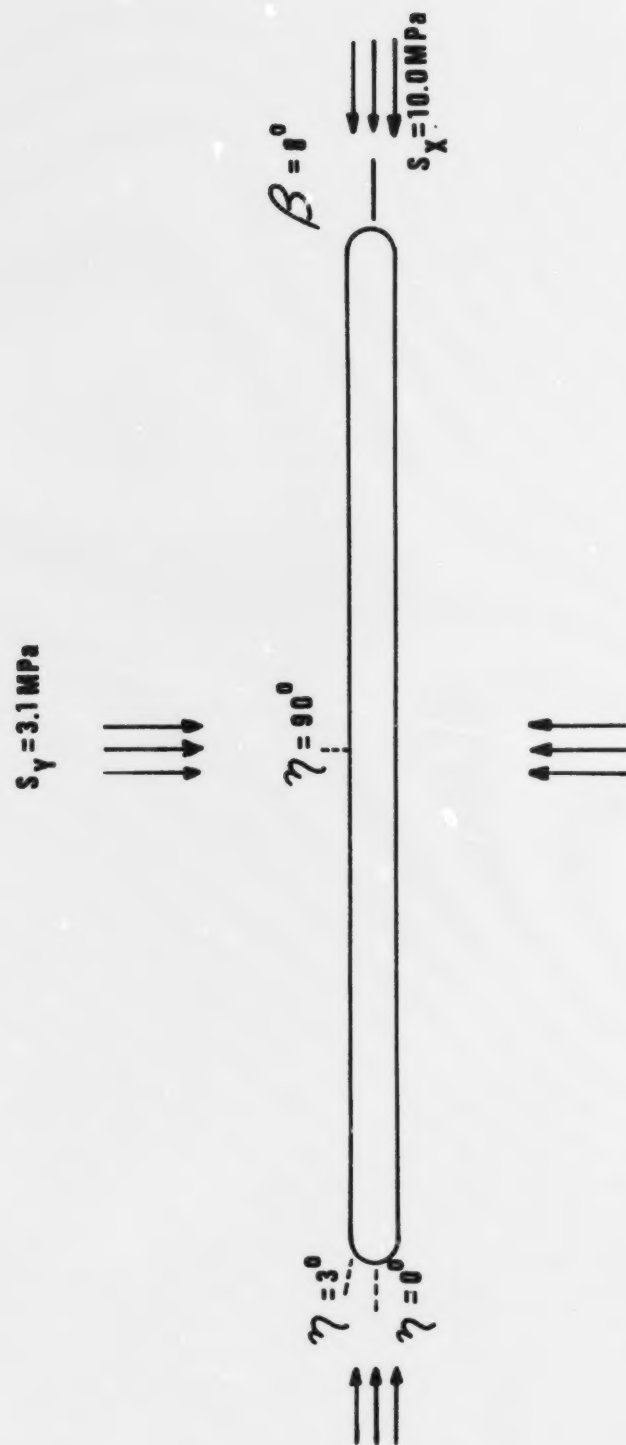


Fig. 11 - Study of stress concentrations on a single opening, Montauban mine, case 1.

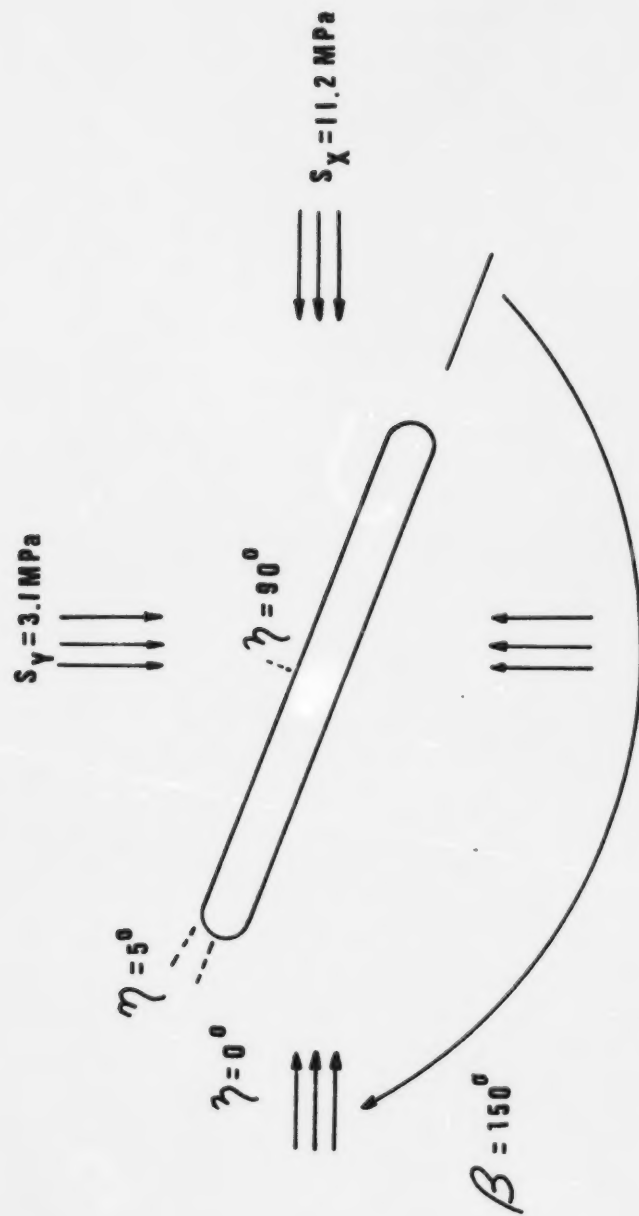


Fig. 12 - Study of stress concentrations on a single opening, Montauban mine, case 2.

Table 1 - Support measures for rock masses of "fair" and "poor" quality.
From Barton et al. (1).

Support category	Q	Conditional factors		P	SPAN/ESR (m)	Type of support	
		RQD/J _n	J _r /J _n	SPAN/ESR	Kg/cm ² (approx.)		
17	10—4	> 30	—	—	1.0	3.5—9	sb (utg)
		≥ 10, ≤ 30	—	—			B (utg) 1—1.5 m
		< 10	—	≥ 6 m			B (utg) 1—1.5 m + S 2—3 cm
		< 10	—	< 6 m			S 2—3 cm
18	10—4	> 5	—	≥ 10 m	1.0	7—15	B (tg) 1—1.5 m + clm
		> 5	—	< 10 m			B (utg) 1—1.5 m + clm
		≤ 5	—	≥ 10 m			B (tg) 1—1.5 m + S 2—3 cm
		≤ 5	—	< 10 m			B (utg) 1—1.5 m + S 2—3 cm
19	10—4	—	—	≥ 20 m	1.0	12—29	B (tg) 1—2 m + S (mr) 10—15 cm
		—	—	< 20 m			B (tg) 1—1.5 m + S (mr) 5—10 cm
20* See note XII	10—4	—	—	≥ 35 m	1.0	24—52	B (tg) 1—2 m + S (mr) 20—25 cm
		—	—	< 35 m			B (tg) 1—2 m + S (mr) 10—20 cm
21	4—1	≥ 12.5	≤ 0.75	—	1.5	2.1—6.5	B (utg) 1 m + S 2—3 cm
		< 12.5	≤ 0.75	—			S 2.5—5 cm
		—	> 0.75	—			B (utg) 1 m
22	4—1	> 10, < 30	> 1.0	—	1.5	4.5—11.5	B (utg) 1 m + clm
		≥ 10	> 1.0	—			S 2.5—7.5 cm
		< 30	≤ 1.0	—			B (utg) 1 m + S (mr) 2.5—5 cm
		≥ 30	—	—			B (utg) 1 m
23	4—1	—	—	≥ 15 m	1.5	8—24	B (tg) 1—1.5 m + S (mr) 10—15 cm
		—	—	< 15 m			B (utg) 1—1.5 m + S (mr) 5—10 m
24* See note XII	4—1	—	—	≥ 30 m	1.5	18—46	B (tg) 1—1.5 m + S (mr) 15—30 cm
		—	—	< 30 m			B (tg) 1—1.5 m + S (mr) 10—15 cm

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

(tg) = tensioned, (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses, see Note XI)

S = shotcrete

(mr) = mesh reinforced

clm = chain link mesh

CCA = cast concrete arch

(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness is given in centimetres (cm).

Table 2 - Stress distribution around two opening settings, Montauban mine.

Case 1

	<u>γ</u>	<u>σ_γ</u>
$S_y = 3.1$ MPa		
$S_x = 10.0$ MPa	0°	117.2 MPa
$\nu = 20$	3°	60.0 MPa
$k = 3.2$	90°	7.8 MPa
$\beta = 0$		
depth = 120 m		

Case 2

	<u>γ</u>	<u>σ_γ</u>
$S_y = 3.1$ MPa		
$S_x = 11.2$ MPa	0°	74.7 MPa
$\nu = 10$	5°	76.9 MPa
$k = 3.6$	90°	8.2 MPa
$\beta = 160$		
depth = 120 m		